UPPER TRISHULI-1 HEP (216MW)			
Client	**Nepal Water and Energy Development Company Private Limited**		
Owner's Engineer	**Tractebel Engineering GmbH / Jade Consult**		
REVIEW NOTE			
Contractor	**Doosan Heavy Industries & Construction Co., Ltd.**		
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The report "Supplementary Investigation Report on Engineering Geology", as the title says, should cover the results of the supplementary investigations, only. Such report does not need – and cannot get - an approval as the data was collected in the field and laboratory without supervision of the OE. The report should serve as input for the designer, who reviews the suggested geotechnical data regarding their reasonability, adjust it, if necessary, and uses the data in the design.

After thorough study of the report in hand it appears that the report is self-reviewing the produced geotechnical data and even conducts design (related issues). There are many, highly sensitive design issues in this geological report addressed, such as "estimation of liquefaction potential of the dam foundation", or "factor of safety calculations for slopes", "factor of safety calculations for wedges in the rock of the Powerhouse". These issues should be addressed in the respective design reports, certainly not in a report on engineering geology.

In the following, all items that refer to design in the report are shaded red. These items should not be included in this report but can be incorporated in respective design reports.

at a depth of 0-7.0m in the upper part of the overburden are relatively loose, while those lower than 7m are medium-dense to dense. The basic value of bearing capacity of the riverbed below 7.0m is >600KPa, and the deformation modulus is >40MPa.

According to the Terzaghi's ultimate bearing capacity formula $P_u = c \cdot N_c + \gamma d N_q + \frac{1}{2} \cdot \gamma b N_r$, a trial calculation is performed to calculate the ultimate bearing capacity of the overburden of the dam foundation in case of shear failure.

It is assumed that the average burial depth of the dam foundation of the project is d=3.0m, the dam foundation width b=10m, the foundation surface is rough, and the floating bulk density of the riverbed overburden r=10.5KN/m^3, the following parameters are used for the trial calculation of the strip dam foundation: C =0, φ=35°.According to Table 1.5-5 below, Nr=45.4, Nq=41.4, Nc=57.7, and the ultimate bearing capacity is calculated to be Pu=3.68Mpa, if the safety factor of the dam is taken K=3, The allowable bearing capacity {P} of the overburden for the dam foundation is 1.2Mpa.

Table 1.4-5 Bearing Capacity Coefficient of Terzaghi Formula

φ	0°	5°	10°	15°	20°	25°	30°	35°	40°	45°
N_r	0	0.51	1.20	1.80	4.0	11.0	21.8	45.4	125	326
N_q	1.0	1.64	2.69	4.45	7.42	12.7	22.5	41.4	81.3	173.3
N_c	5.71	7.32	9.58	12.9	17.6	25.1	37.2	57.7	95.7	172.2

1.4.2.3 Evaluation on Foundation Soils against Liquefaction

The PGA of the bedrock with a 500-yr return period in the project area is 0.25-0.30g, and the corresponding seismic intensity is Mag. VIII. The sand layer (lens) has the seismic conditions for vibration liquefaction.

In the BDH-3 borehole, the medium-coarse sand lens are located at a depth of 10.45-11.0m and 17.0-17.4m, and the corresponding N63.5 values are 29 and 30 blows respectively. According to *Code for Seismic Design of Buildings (GB50011-2010)*, at a

depth of 20m below the ground, the critical SPT value for liquefaction criterion may be calculated by the following formula:

$$N_{cr} = N_0 \beta [\ln(0.6 d_s + 1.5) - 0.1 d_w] \sqrt{\frac{3}{\rho_c}}$$

Where,

N_{cr} - critical SPT value for liquefaction criterion;

N_0 - reference SPT value for liquefaction criterion, adopted as per Table 3.4-6;

ds - depth at which SPT is performed for saturated soil (m);

dw - groundwater level (m);

ρ_c - percentage of clay content, when it is less than 3 or sandy soils, taking 3;

β - adjustment factor, 0.8 for 1st group of design earthquakes, 0.95 for 2nd group, and 1.05 for 3rd group.

Table 1.4-6 SPT Blows for Liquefaction Criteria

Design basic seismic acceleration (g)	0.1	0.15	0.2	0.3	0.4
Referenced SPT blows for liquefaction criteria	7	10	12	16	19

In connection with the project, when the PGA for OBE case (approximately at 145-yr frequency) is 0.19g, taking N_0=12, ds=10.5m and 17.2m, dw=0m, ρ_c=3, and β=1.05 (maximum value), and the critical SPT value N_{cr} for liquefaction criterion is 20 and 21 respectively. Since N = (29 and 30)> (20 and 21), it may be judged that sand liquefaction will not occur.

However, when the PGA=0.83g at MCE case, N_0=19, ds=10.5m and 17.2m, dw=0m, ρ_c=3, and β=1.05 (maximum value), and the critical SPT value N_{cr} for liquefaction criterion is 66 and 76 respectively. Since N = (29 and 30) < N_{cr}, it may be judged that the sand will be liquefied. It is suggested that during the construction and excavation of

the dam foundation, the upper sand specimen should be taken to conduct geotechnical tests such as sieve and relative density tests for further review of the above conclusions by analogy.

1.4.2.4 Evaluation on Permeability of Foundation Soils

At previous stage, DKJV has carried out a large number of water injection tests with constant head for the riverbed overburden (see Table 1.4-7). Therefore, the supplementary investigation did not specifically focus on the seepage test for the riverbed overburden. According to the analysis of the seepage results of DKJV, the permeability coefficient for riverbed overburden $K=2.6\times10^{-2}$cm/s - 2.5×10^{-3}cm/s, which indicates that the overburden is featured by medium to strong water permeability. In the supplementary investigation, double plugs were used to conduct 16-stage water pressure tests on the bedrock sections. The results are shown in Table 1.4-8 from which it can be seen that the permeability coefficient K of the bedrock at the dam site is $K=1.12\times10^{-4}$cm/s-4.41×10^{-5}cm/s, which indicates that the bedrock is generally weakly to slightly permeable.

filled with quartz, detritus, etc., straight and slightly rough on the surface, with good cementation; (2) Joint J2: attitude 240-250°/60°-70°, spacing 1.0m-2.0m, developed discontinuously, straight and rough on the joint surface, filled with detritus, with medium cementation, (3) Joint J3: attitude163°/63°, and the properties are similar to J2.

The thickness of the overlying rock mass in the main area of the caverns is 220m~320m, according to the rock mass density of 2.70g/cm^3 and the rock mass stress σh=γH, the gravity stress is calculated to be 5.9MPa~8.6MPa. Considering the other effects such as tectonic stress, temperature stress, and metamorphism, the initial maximum principal stress may be >10.0MPa. It is preliminarily judged that the initial in-situ stress of the powerhouse is in a low-medium in-situ stress zone. A slight rock burst may occur following the excavation of the tunnel, which may have slight impact on the normal construction.

Up to now, the ground stress testing has not been carried out, and the specific ground stress level and orientation have not been obtained in detail.

According to the analysis of the origin of the Qinghai-Tibet Plateau, the modern crustal movement in the project area is still dominated by the horizontal thrust movement of the Indian Ocean plate against the Asia-Europe plate. It is preliminarily judged that the axis of the principal compressive stress is at 0-45°, which is the horizontal compression stress in the NNE direction. By means of UNWEDGE analysis software, the calculation parameters in Table 3.5-1 are applied to preliminarily search the block stability of cavern, as shown in Table 1.5-2.

Table 1.5-1 Discontinuity Parameters Applied for UNWEDGE Software Analysis

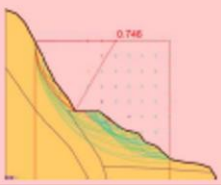
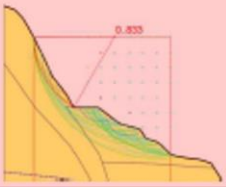
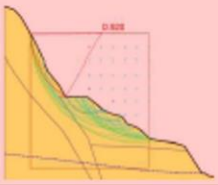
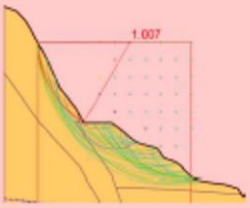
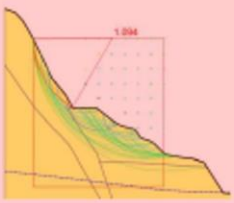
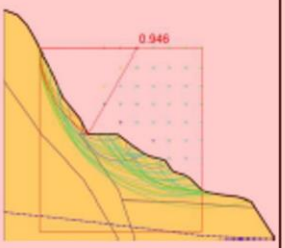
Properties of joint	Coarseness of joint JRC	Rock strength on both sides of joint JCS (MPa)	C (MPa)	Φ (°)	Underground water pressure (MPa)
Hard discontinuity (bedding plane J1)	6	60	0.20	35	0.05
Debris type of rock block J2 and J3	8	50	0.15	28.8	0.05

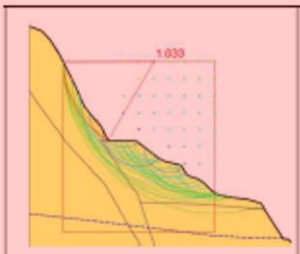
14.

(2) It is assumed that the slope of the overburden layer lower than the road in Section C-C is in a stable state (Fs≥1.2). Stress analysis method: total stress method. Calculation method: simplified BISHOP.

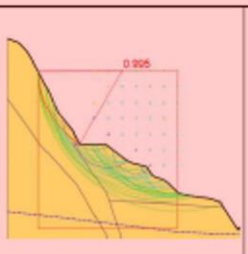
Method of review: Swedish Slice Method. The bulk density of the slope overburden higher than the road is taken 21.5KN/m^3. The inversion results are shown in Table 1.4-15.

Table 1.4-14 Summary of Slope Inversions above the Road

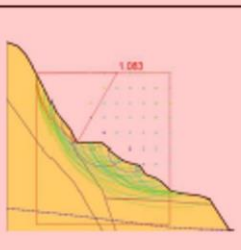
1. C=10, φ=35°	2. C=15, φ=35°	3. C=20, φ=35°
		
Fs=0.746, not conform to actual situation	Fs=0.833, not conform to actual situation	Fs=0.920, not conform to actual situation
4. C=25, φ=35°	5. C=30, φ=35°	6. C=25, φ=32°
		
Fs=1.007, conform to actual situation	Fs=1.094, conform to actual situation	Fs=0.946, conform to actual situation
7. C=30, φ=32°	8. C=30, φ=30°	9. C=35, φ=30°



Fs=1.033, conform to actual situation

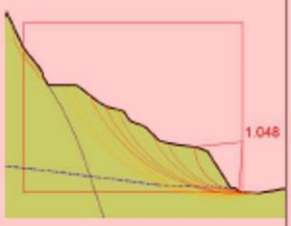
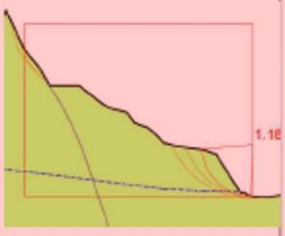
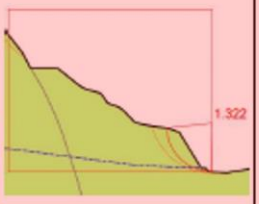
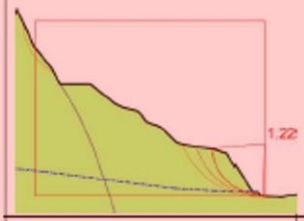
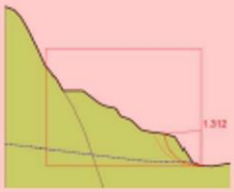
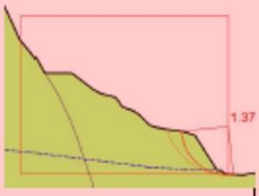
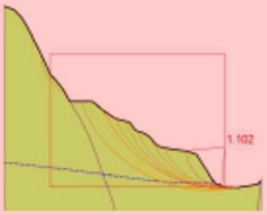
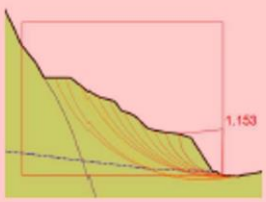
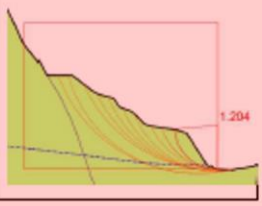


Fs=0.995, conform to actual situation



Fs=1.083, conform to actual situation

Table 1.4-15 Summary of Slope Inversions below the Road

1. C=25, φ=30°	2. C=25, φ=35°	3. C=25, φ=40°
		
Fs=1.048	Fs=1.18	Fs=1.322
4. C=30, φ=35°	5. C=30, φ=38°	6. C=30, φ=40°
		
Fs=1.229	Fs=1.312	Fs=1.371
7. C=30, φ=30°	8. C=35, φ=30°	9. C=40, φ=30°
		

31

Fs=1.102	Fs=1.153	Fs=1.204

In accordance with summary of inversion results in Tables 1.4-14 and 1.4-15, the recommended mechanical parameters of bank slope overburden of apron area in stilling basin for this supplementary investigation are shown in Table 1.4-16.

Table 1.4-16 Recommended Mechanical Parameters of Bank Slope Overburden in Apron Area

	Classification	ρ (g/cm³)	Φ (°)	C (KPa)
Military road slope	Colluvial gravelly soil (above the road)	1.9	32	25
	Dilluvial-mixed block and gravelly soil (below the road)	2.1	35	30

1.5 Geological Conditions of Headrace and Power Generation System and Evaluation

The headrace and power generation system is arranged on the right bank of the Trishuli River, where the bank slopes are high and steep and the mountains are massive. The tunnel mainly runs through the medium-hard to hard schist and gneiss rock formations. The rock formations dip to the bank and have a gentle dip angle. The headrace and power generation system are deeply seated below the ground, with a general burial depth greater than 150m (the shallowest 75m, and the deepest 605m). The tunnel mainly spans two large gullies with flowing water. The bedrock is exposed in the bottom of the gullies exactly higher than tunnel, and the overlying rock is greater than 100m deep. In general, the geological conditions along the headrace and power generation system are relatively good.

The headrace and power generation system are about 10.7km long, composed of the power intake, underground desander basin, upper horizontal section of headrace tunnel, impedance surge tank, pressure shaft, lower horizontal section, underground powerhouse, tailrace tunnel, etc.

filled with quartz, detritus, etc., straight and slightly rough on the surface, with good cementation; (2) Joint J2: attitude 240-250°/60°-70°, spacing 1.0m-2.0m, developed discontinuously, straight and rough on the joint surface, filled with detritus, with medium cementation, (3) Joint J3: attitude163°/63°, and the properties are similar to J2.

The thickness of the overlying rock mass in the main area of the caverns is 220m~320m, according to the rock mass density of 2.70g/cm^3 and the rock mass stress σh=γH, the gravity stress is calculated to be 5.9MPa~8.6MPa. Considering the other effects such as tectonic stress, temperature stress, and metamorphism, the initial maximum principal stress may be >10.0MPa. It is preliminarily judged that the initial in-situ stress of the powerhouse is in a low-medium in-situ stress zone. A slight rock burst may occur following the excavation of the tunnel, which may have slight impact on the normal construction.

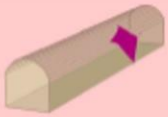
Up to now, the ground stress testing has not been carried out, and the specific ground stress level and orientation have not been obtained in detail.

According to the analysis of the origin of the Qinghai-Tibet Plateau, the modern crustal movement in the project area is still dominated by the horizontal thrust movement of the Indian Ocean plate against the Asia-Europe plate. It is preliminarily judged that the axis of the principal compressive stress is at 0-45°, which is the horizontal compression stress in the NNE direction. By means of UNWEDGE analysis software, the calculation parameters in Table 3.5-1 are applied to preliminarily search the block stability of cavern, as shown in Table 1.5-2.

Table 1.5-1 Discontinuity Parameters Applied for UNWEDGE Software Analysis

Properties of joint	Coarseness of joint JRC	Rock strength on both sides of joint JCS (MPa)	C (MPa)	Φ (°)	Underground water pressure (MPa)
Hard discontinuity (bedding plane J1)	6	60	0.20	35	0.05
Debris type of rock block J2 and J3	8	50	0.15	28.8	0.05

Table 1.5-2 Instability Block Search Statistics of Cavern by UNWEDGE Software

S.N.	Fissure set number	Block volume (m^3)	Maximum block height (m)	Safety factor	Development position	Instability mode	Graphical presentation of block (cavern)
1	J1/J2	39.18	2.81	0.068	outer wall	slip	
2	Slip along J2	23.703	2.35	0.431	crown	slip	
3	J1/J2/J3	99.571	3.72	0	right crown	fall	

It is preliminarily judged that at deep locations below the ground, the discontinuities in the caverns should be mainly closed or slightly open rigid discontinuities. The tunnel orientation of the chamber section is 233°, the rock strike is 240-250°, and the direction of the bedding plane intersects the tunnel axis at a small angle. The stability analysis of the cavern blocks initially shows that the rocks of desander basin caverns are cut by the joints, mostly shown as rockfall and sliding. The volume of the unstable block is about 0.3-100m^3, and the height of the block is about 0.4-3.7m. During the excavation process, a reasonable excavation sequence should be adopted for layered excavation. A reasonable height for the layered excavation may avoid large potentially unstable blocks. For the excavated caverns, monitoring and normal geological inspections shall be conducted, and unstable blocks shall be reinforced in time.

It is preliminarily judged that the possibility of overall instability of the desander basin is small. The deformation and failure pattern of the surrounding rock of the cavern are mainly collapse of the cavern crowns, wedge collapse at the intersection of the caverns, and the rockfall blocks, and the possibility of failure of the side walls is small.

arranged at the bottom of the lower surge tank and is connected vertically to the headrace tunnel. The inner diameter of the lower surge tank is 8.5m, the floor elevation of the upper surge tank is EL.1255.0m, the crown elevation is EL.1268.5m, and the total height is 73.186m. There is an access tunnel (ventilation tunnel) on the upstream side of the surge tank with a diameter of 5m×5m.

The underground surge tank is located in the mountain on the right bank, with poor surface topography. There are small gullies on both sides. The topography on the river side is steep, and the surface is covered by colluvial rocks and gravels. According to drill holes provided by DKJV, the average burial depth of the crown of the impedance surge tank is about 85m, of which the thickness of the overburden is about 5.0m, the average thickness of the overlying rock mass is about 80m, and the horizontal burial depth is about 110m. The lithology is dominated by medium-thick to thick quartz gneiss, with mica schist interbedded locally, which is hard rock in general. The rock formation has a dip of NW~NE and a dip angle of 20°~30°. The surrounding rock mass of the entire shaft is mainly classified as Class III. The groundwater type is mainly bedrock fissure water. The groundwater activity is weak, will be mainly characterized by dry walls, wetness in scattered areas or seepage or dripping along discontinuities such as joints, fissures and faults.

Seen from the surface, the following discontinuities are mainly developed in the surge tank area: (1) J1 (bedding plane), attitude (dip/dip angle): 330-340°/20-30°, spacing 20cm-50cm, filled with quartz, detritus, etc., straight and slightly rough on the surface, with good cementation; (2) Joint J2: 260°/65°, spacing 1.0-2.0m, discontinuously developed, straight and rough on the joint surface, filled with detritus, with medium cementation . (3) Joint J3 (cutting fissures): 312°/74°, similar properties compared to J2.

By means of UNWEDGE analysis software, the calculation parameters in Table 1.5-4 are applied to search the block stability of surge tank, as shown in Table 1.5-5.

Table 1.5-4 Discontinuity Parameters Applied for UNWEDGE Software Analysis

Properties of joint	Coarseness of joint JRC	Rock strength on both sides of joint JCS (MPa)	C (MPa)	Φ (°)	Underground water pressure (MPa)
Hard discontinuity (bedding plane)	6	70	0.20	35	0
Rock debris type of rock block J2 and J3	8	50	0.15	28.8	0

Table 1.5-5 Instability Block Search Statistics of Surge tank by UNWEDGE Software

S.N.	Fissure set number	Block volume (m^3)	Maximum block height (m)	Safety factor	Development position	Instability mode	Graphical presentation of block
1	J1/J2/J3, slip along J3	3.606	1.10	0.597	wall of upper surge tank on southeast side	slip	
2	J1/J2/J3, slip along J2	23.157	2.55	0.729	Wall of upper surge tank on east side	slip	
3	J1/J2/J3, slip along J2	0.704	0.79	0.861	Wall of lower surge tank on east side	slip	
4	J2/J3	2.408	1.39	0.878	Wall of lower surge tank on northeast side	slip	

5	J1/J2/J3, slip along J3	0.108	0.34	0.727	Wall of lower surge tank on southeast side	slip	
6	Slip along J3	315.5	6.05	0.416	Crown of surge tank	fall	

The preliminary analysis of the block stability of the caverns shows that there are unstable wedges on the wall of the surge tank due to the cutting of the joints and fissures, and the main pattern of block instability is slipping. The volume of the unstable block is about 0.1-23.1m^3. The height of the blocks is about 0.3-2.6m. The blocks are mainly distributed on the east, southeast and northeast side of the shaft walls. There may also be potential massive unstable blocks on the crown of the surge tank with a maximum volume of about 315.5m^3. During the excavation process, a reasonable layered excavation sequence shall be adopted. A reasonable height for the layered excavation may avoid large potentially unstable blocks. For the excavated caverns, monitoring and normal routine geological inspections shall be conducted, and initial support shall be provided for the excavated tunnel section in time.

1.5.5 Pressure Shaft

The depth of the pressure shaft is about 300m, and the excavation diameter of the shaft is 6.7m. According to the geological survey, there may be a lithological boundary between schist and gneiss in the shaft. It is inferred that the lithology at 0-260m of the shaft section is gneiss, which is hard rock, and 260-300m is schist, which is medium-hard rock. The rock mass in the whole shaft is slightly weathered or fresh rock mass, and the strike of the lithology is almost perpendicular to the shaft axis, and the attitude of the formation is favorable to the stability of the shaft walls.

The discontinuities developed in the shaft are similar to surge tank, with three main sets: (1) J1 (bedding plane), attitude (dip/dip angle): 330~340°/20-30°, spacing 20cm~50cm,

may be >10.0MPa. In general, the initial in-situ stress in the powerhouse area is preliminarily classified as low-medium in-situ stress area, and the principal compressive stress is the horizontal compression stress developed in NNE.

It is judged that in the deep underground, the discontinuities in the caverns should be mainly closed or slightly open rigid discontinuities. The orientation along the long axis of the main powerhouse and main transformer caverns is 93°, where the strike of rock formations is 65°, indicating that the intersection angle between them is 28°.

By means of UNWEDGE analysis software, the calculation parameters in Table 1.6-1 are applied to search the block stability of surge tank, as shown in Table 1.6-2.

Table 1.6-1 Discontinuity Parameters Applied for UNWEDGE Software Analysis

Properties of joint	Coarseness of joint JRC	Rock strength on both sides of joint JCS (MPa)	C (MPa)	Φ (°)	Underground water pressure (MPa)
Hard discontinuity (bedding plane)	6-8	60	0.20	35	0.1
Rock debris type of rock block J2-J6	8-10	45	0.15	28.8	0.1

Table 1.6-2 Instability Block Search Statistics of Underground Powerhouse by UNWEDGE Software

S.N.	Fissure set number	Block volume (m^3)	Maximum block height (m)	Safety factor	Development position	Instability mode	Graphical presentation of block
1	J1/J2/J4	0.539	0.35	0	crown	fall	

2	J1/J2/J4	13.693	1.52	0.296	crown	fall	
3	J1/J2/J5	2.236	0.67	0	crown	fall	
4	J1/J2/J6	0.335	0.33	0.237	crown	fall	
5	J1/J2/J6	0.546	0.44	0	crown	fall	
6	J1/J3/J4	0.217	0.227	0.25	crown	fall	
7	J1/J3/J4	0.206	0.25	0	crown	fall	

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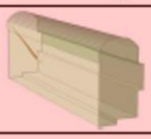
8	J1/J3/J5, slip along J5	1.141	0.55	0.291	crown	fall	
9	J1/J3/J5, slip along intersecti on line	8.861	1.41	0	crown	fall	
10	J1/J3/J5, slip along intersecti on line	7.896	1.0	0	running through side wall on inner side	slip	
11	J3/J6	0.136	0.24	0.392	crown	slip	
12	J1/J3/J6	45.575	3.56	0	crown	fall	
13	J2/J3/J4	0.674	0.47	0	crown	fall	
14	J2/J3/J4	9.344	1.6	0	crown	fall	
15	J2/J3/J5 , slip along J2	8.407	1.03	0.415	upstrea m end wall	slip	

16	J3/J4/J5	1.822	0.76	0	crown	fall	
17	J3/J4/J5, slip along J4	7.27	0.99	0.154	downstream end wall	slip	
18	J3/J4/J6, slip along J3	0.124	0.24	0.142	crown	slip	
19	J3/J4/J6	6.362	1.51	0	crown	fall	
20	J4/J5/J6, slip along J6	1.555	0.88	0.257	crown	slip	

The preliminary analysis of the block stability of the caverns shows that there will be several unstable wedges in the crown of the powerhouse due to the cutting of the joints and fissures. The main pattern of block instability is rockfall from crown areas. There will also be unstable blocks on the local side walls, and the main pattern is the slipping of wedges. The volume of the unstable block is about 0.1-45.6m^3, and the height of the block is about 0.2-3.6m. During the excavation process, a reasonable layered excavation sequence shall be used and a reasonable excavation height to avoid large potential unstable blocks. Routine monitoring/observation and normal geological inspections shall be made for the excavated caverns, and the initial support of the excavated section shall be provided in a timely manner.

At this stage, only rough analysis can be conducted based on the discontinuity exposed on the ground surface. At a later stage, the preliminary construction of ventilation tunnel and construction adits should be combined to collect as much geological data as possible to analyze the joint development pattern for the main areas of the caverns

the project is set as 60MPa. On this basis, ROCKDATE software is used to perform trial calculations for the rock mass parameters by GSI method of Hook-Brown failure criterion, with the detailed trial calculation results shown in Table 1.8-3.

The comprehensive DKJV investigation results are shown in Table 1.8-1 and the calculation results by Hoek-Brown failure criterion are shown in Table 1.8-3. In combination with similar engineering experiences (Upper Trishuli 3A), the recommended parameters of surrounding rocks of underground caverns for this supplementary investigation are shown in Table 1.8-4.

Table 1.8-3 Tentative Calculation of Rock Mass Parameters by GSI Method as per Hoek-Brown Failure Criterion

Rock class	Unit weight	UCS r	gci	GSI	mi	D	mb	s	a	μ	H	Hoek-Brown Result			
(Q)	P (g/cm³)	(MPa)										Em (GPa)	c (MPa)	φ (°)	UCSrm (MPa)
Ⅰ (Q > 40)	2.7	60	60									58.0	4.2	52.3	26.1
Ⅱ (10-40)	2.7	60	60									27.5	2.3	51.3	12.6
Ⅲa (7-10)	2.65	60	60									10.3	1.20	49.1	4.8
Ⅲb (4-7)	2.65	60	60	45	14	0	1.964	0.0022	0.51	0.25	≥150	5.8	0.83	48.5	2.7
Ⅳ (1-4)	2.6	60	60	30	14	0	1.149	0.0004	0.52	0.35	≤150	2.4	0.59	43.9	1.0
Ⅴ (Q < 1)	2.45	50	60	18	14	0	0.695	0.0001	0.55	0.40	≤100	1.2	0.31	42.4	0.4

Suggested and approved rock parameters can be found in respective document.

In the table: gci refers to uniaxial compressive strength of undisturbed rock block; GSI refers to integrity of rock mass; mi refers to lithological index; D refers to disturbed parameter Mb of blasting effect and stress release; a refers to empirical parameter of different dimension 1 of different rock masses; s refers to fracture index of rock mass; H refers to the embedded depth of distribution location of surrounding rock.

Table 1.8-4 Recommended Rock Mass Parameters for Underground Caverns after This Investigation

Rock type	Rock class	Q	γ (kN/m³)	μ	Em (GPa)	C (MPa)	Φ (°)	Φr (°)	Cr (MPa)	UCS rm (MPa)	Intact UCS r (MPa)
Schist/ gneiss	I	> 40	27.0	0.20	30.0	5.0	50	42	1.80	27.5	60
	II	10--40	27.0	0.25	20.5	2.5	48	38	0.95	13.0	60
	IIIa	7--10	26.5	0.25	10.0	1.0	45	35	0.35	4.8	60
	IIIb	4--7	26.5	0.28	5.0	0.8	42	33	0.30	3.6	60
	IV	1--4	26.0	0.30	2.3	0.5	35	27.5	0.15	1.9	60
	V	< 1								0.6	60

Suggested and approved rock parameters can be found in respective document.

Remark: In ... rock mass. In accordan... ...al internal frictional angle Φr is reduced as per 0.7-0.8 time of peak strength and the residual cohesion Cr is reduced as per 0.3 time of peak strength.

1.9 Construction Materials

According to the investigation reports collected at the FS stage and the basic design stage (main report at FS stage and geological investigation report at basic design stage) and the attached drawings (figures), pit exploration and lab test were made covering soil materials, riverbed natural sand and gravel materials for the building materials of the project. In this supplementary investigation report, no further investigation of building materials is performed. The quartz schist and gneiss excavated may also be used as one of the sources of concrete aggregates, but the mica schist in the excavated material needs to be removed, and it is suggested that the alkali activity test of the aggregate be carried out before the use of excavated materials.

1.10 Conclusions and Suggestions

(1) The project area is located in the Sub Himalayas Tectonic Belt at the southern foot of the Himalayas in Nepal, which is featured by poor tectonic stability and strong seismic activity. According to DETERMINISTIC SEISMIC HAZARD ANALYSIS REPORT, the PGA of the Operating Basic Earthquake (OBE) of the project (with a 145-yr return period approximately) is 0.19g; the PGA of the Design Basis Earthquake (DBE) (with a 475-yr return period approximately) is 0.39g; the PGA of Maximum Credible Earthquake (MCE) (with a 3,000-yr return period approximately) is 0.83g.

(2) A small reservoir will be formed after impoundment. Except for the 100m-long bank slope along the road on the left bank upstream of the dam, which needs to be reinforced and protected, the geological problem induced by the reservoir impoundment is slight. In general, the engineering geological conditions of the reservoir are good.

(3) The river channel at the dam site is narrow, the bedrock slopes on both banks are high and steep, and the bank slopes are stable at present. The depth of boulder and pebble deposit in the riverbed is about 45m at the deepest. The composition is mainly large blocks, boulders and coarse-grained pebbles/gravels. The pores of the boulder skeletons are filled with medium sands. The boreholes did not reveal the continuous sand distribution in the riverbed overburden. In general, as the foundation is medium dense-dense, and the bearing capacity can meet the requirements of dam foundation, it can be used as the natural foundation of the dam, but the super-large blocks and boulders in the overburden are unevenly distributed, especially at the sluice piers and guide walls areas, where the foundation soils may experience uneven settlement. Under the Operating Basic Earthquake (OBE) condition, the sand lens in the dam foundation soil will not cause sand liquefaction, but under the maximum credible earthquake (MCE) condition (PGA=0.83g), the sand lens may be liquefied. In the case of abnormally high seismic stability standards of the dam, it is suggested to take remedial measures (such as dynamic compaction and consolidation grouting, etc.) within a certain depth of the

dam foundation, especially for the pier foundation, to improve the uniformity of the foundation soils. If sand layer, silty soil and other local adverse formations are exposed on the foundation surface, all of them shall be removed and replaced; the boulders and pebble layers in the riverbed have medium to strong water permeability, for which anti-seepage drainage facilities shall be provided; the bedrock on both banks is weakly to slightly permeable in general, and the problem of bypass seepage is not prominent. The seepage in bank slopes can be controlled in combination with the connecting structures on both banks.

(4) The surrounding rock masses of the underground caverns of the project is mainly composed of quartz schist, mica schist and quartz gneiss, in medium-thick to thick bedded structure, medium-hard to hard in general. The supplementary exploratory adit is not completed as planned, and most of the information available come from the documents of previous investigations. Only one 150m-long exploratory adit was completed in the earlier exploration stage. With reference to the previous exploration results and in combination with the supplementary geological mapping of the ground surface, predictions and judgments on the geological conditions of underground caverns are made in this supplementary investigation report.

According to preliminary judgment, the surrounding rock of the underground caverns in the project area is moderate to good as a whole, and the conditions are favorable for caverns. The groundwater activity is not strong, the rock burst problem is slight, and the possibility of large-scale underground cavern instability is quite small. It is suggested that at the next stage, in combination with the layout of the original exploratory adit and the actual excavation progress of construction adits, to collect ASAP and as much as possible the geological data of the excavated caverns, and compile and update the latest investigation results in a timely manner.

(5) During the construction, the conditions of the suspected shear zone at section (Ch. 2+500) of the headrace tunnel across the gully shall be further checked; the surrounding

DISCLAIMER

Any approval will not release the Contractor from any of his obligations under the Contract.

Dr. Ulrich Glawe

(Signature)